# Dynamic wave force of tsunamis acting on a structure

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**Abstract.** This paper aims to measure the wave force of tsunamis acting on prevention structures along the coast such as seawalls and breakwaters, in the hydraulic experiment, in order to revise the existing wave force formula used for the design in the past, by introducing four types of wave pressure: dynamic, sustained, impact standing, and overflowing. Impact standing wave pressure is observed in the collision of reflected and incident waves. Overflowing wave pressure occurs when waves running over a structure collide on the back. The values of these wave forces are found to be more significant than the dynamic and sustained pressures. The formulation to estimate each wave force for design of a coastal structure is proposed.

## 1. Introduction

Disaster prevention work/research in Japan has been energetically carried out since the 1960 Chilean tsunami attacked the coastal area. Although numerous seawalls of 5–6 m in height were constructed by referring the measured wave heights of the 1960 tsunami, only static pressure was taken into consideration for a design of structures, meaning that the design method does not include any dynamic pressure that is significant and not negligible in the case of a tsunami attack. Generally, tsunamis are amplified in a shallow sea region, and occasionally transfer to bore after the break. An experimental study by Fukui *et al.* (1962a, b) pointed out that the hydrodynamic pressure of the tsunamis, especially bore-type, becomes larger than one in gravity wave. We, therefore, have some concerns and anxiety as to whether or not the existing structure system along the coast is strong enough against a tsunami attack with a strong dynamic wave force, although some safety factors were taken into the design according to engineering experience.

Moreover, a tsunami disaster prevention manual agreed upon by two governmental organizations in Japan was established in 1983. In 1997 the manual was revised on an agreement among seven organizations that are responsible for tsunami disaster prevention from different points of view. Since then, not only structures but also human-action-based mitigation methods have been taken into consideration, and it is stated that a structure may not perfectly prevent a tsunami attack, and a flow over a structure and flood of the area behind it would be expected. New damage would happen due to overflow, eroding the foundation and destroying the wall by dynamic force. A study on tsunamis overflowing a structure is required for creating scenarios or a mitigation plan.

Goda's formula, which is widely used in the present design of structure, was proposed in 1973 and is a well-known formula used to estimate the pressure of gravity waves acting on a coastal structure (Goda, 1973). However, the formula for the design of a tsunami-protective structure is still limited

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to the use of Goda's formula. Up to now, there is little research on the wave force of a tsunami on a structure; Fukui *et al.* (1962a) measured water level and wave pressure in detail, and proposed the equation to evaluate wave celerity of the bore. Fukui *et al.* (1962b) also studied run-up height, standing wave height, wave pressure, and overflowing quantity to estimate the wave force for the design of structure. Matsutomi (1991) measured the pressure of the bore in detail in hydraulic experiments and clarified the characteristics of a tsunami bore. However, design of structure is limited to a simple rectangular shape and there are no studies on interactive behaviors between a tsunami and a type of structure.

The aim of the present study is to carry out the hydraulic experiments with the transient wave-like bore by using the experiment open-channel, and to measure the wave force of a bore acting on the structures by using a newly developed and highly accurate sensor system. Using the data measured in this study, we try to improve the formula for estimating the wave force.

## 2. Hydraulic Experiment

### 2.1 Experimental setup and condition

Figure 1 shows the experimental setup of a one-dimensional open channel of 12 m in length, 0.3 m in width, and 0.44 m in height. A paddle installed at one end can generate a bore-type transient wave. We carry out two different experiments: non-overflowing model (case 1) and overflowing model (case 2).

In case 1, the slope modeling a coastal structure such as a sea wall or breakwater is made at the other end as shown in Fig. 1a. The pressure gauge of 1 cm in diameter and 9800 Pa in pressure capacity is put on the slope with a spacing of 1 cm in order to measure a profile of the wave pressure on the structure. Three water leveling gauges are installed at the front of the wave paddle side, at the center of the channel, and near the sloping board side, and one current meter to measure flow velocity is placed near gauge 2 in Fig. 1a. Experimental conditions are selected with different slopes and water depths—1:1.53, 1.3, 1, 0.6, 0.3, and 0.0 of the slope, and 5, 10, and 15 cm of the water depth. The axis of $X$ with the origin at the bottom of the slope of the structure is introduced for the reference of the measurement.

In case 2, the trapezoidal structure modeling the seawall at Taro-cho, Iwate Prefecture in Japan, is made at the other end as shown in Fig. 1b. A pressure gauge of the same diameter and pressure capacity is put on the crest, back slope, and area behind the structure with spacing of ~2–4 cm to measure a profile of the wave pressure on those areas. Three wave gauges are installed at the front of the wave paddle side, at the center of the channel, and near the model side, as well as one gauge and current meter to measure overflowing discharge on the center of the crest. Experimental conditions are selected with different back slopes and water depths—1:1.5, 1.25, and 1 of the back slope, and 10, 12, and 15 cm of the water depth. The origin of the x-axis is shifted from the bottom to the top of the structure.
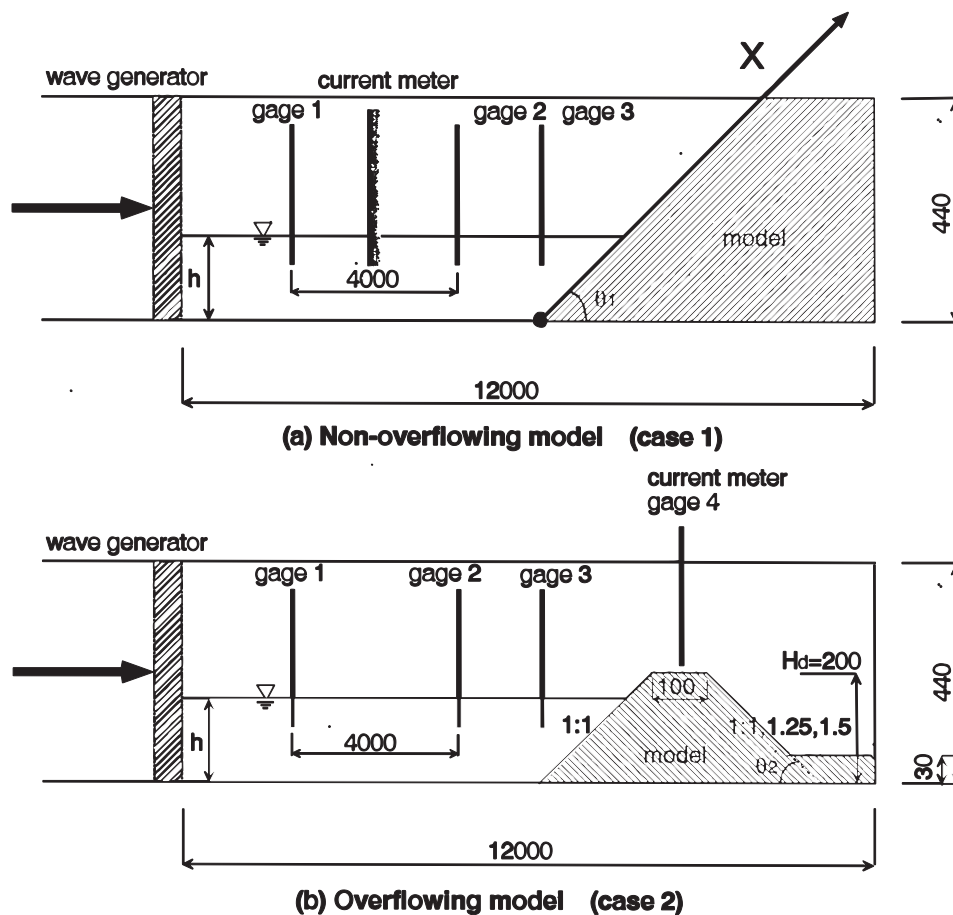
**Figure 1**: Experimental setup (units in mm).

Selecting the time interval, t, is important to measure instantaneous hydraulic force. The longer the time interval selected, the smaller is the value of the peak wave force measured due to the low sampling data. Considering the time period of the peak, we assume four different time intervals to measure wave pressure under the same condition. It is suggested that the peak of pressure appears clearly as long as the time interval is smaller than 0.002 s. However, the data contains much noise in the case of t = 0.001 s. Therefore, we selected t = 0.002 s as the best time interval.

## 2.2   Experimental measurement and its results

The time history of wave pressure at the point of $x = 2$ cm, at the level of still water for case 1 is shown in Fig. 2, suggesting the three different peaks in wave pressure acting on a structure. The first is called *dynamic wave pressure* that appears when the incident wave reaches a slope and impacts it. The second is *sustained wave pressure* that is observed when the wave run-up and water level remains high through a series of incident waves. The last is *wave pressure* that instantaneously appears when reflected and
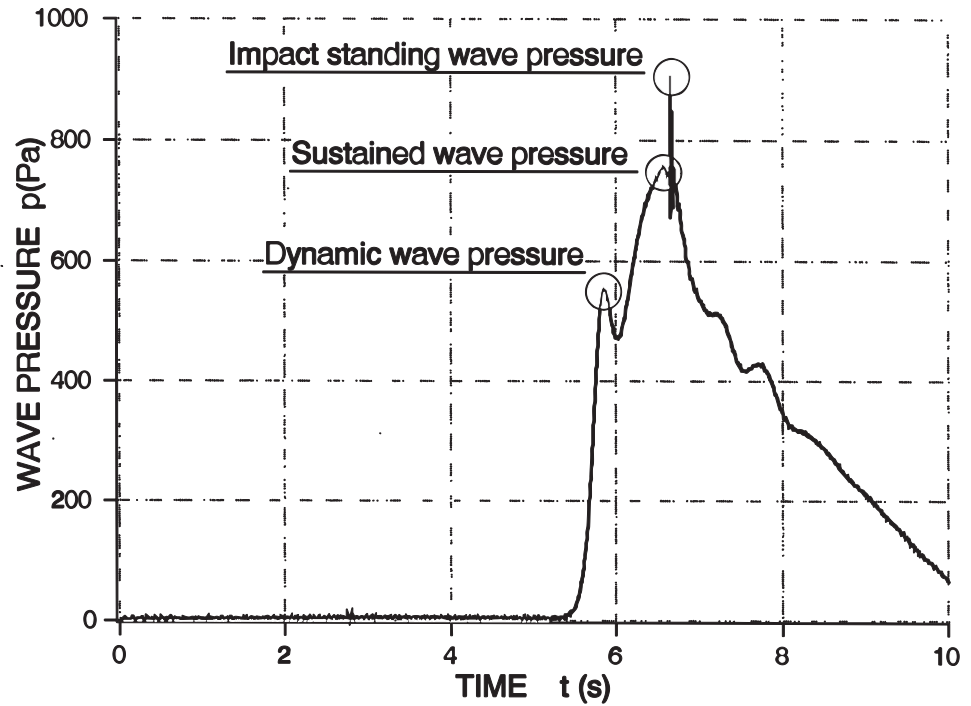
**Figure 2**: Wave pressure variation with time (slope 1:1.53, $h = 10$ cm, $x = 2$ cm).

incident waves collide. The last has never been reported in previous studies, but the wavelength is longer than the run-up length on the slope. We named it *impact standing wave pressure.*

The overflowing wave pressure acting on the crest, the back slope of the model, and the back area with different water depth in the example of the back slope 1:1 for case 2, is shown in Fig. 3. In this example, the remarkably large value is measured at the point of the back area near the model. However, we have not observed the maximum overflowing wave pressure clearly in the back slope 1:1.5. This suggests that the example with deeper water depth and steeper slope makes a larger collision that occurs when the overflowing wave reaches the back area. Since the present study shows that rather local and large force as the impact standing, and overflowing wave force acts on the structure and its back area, respectively, in a short time, this should be carefully considered and estimated for the design of the structure.
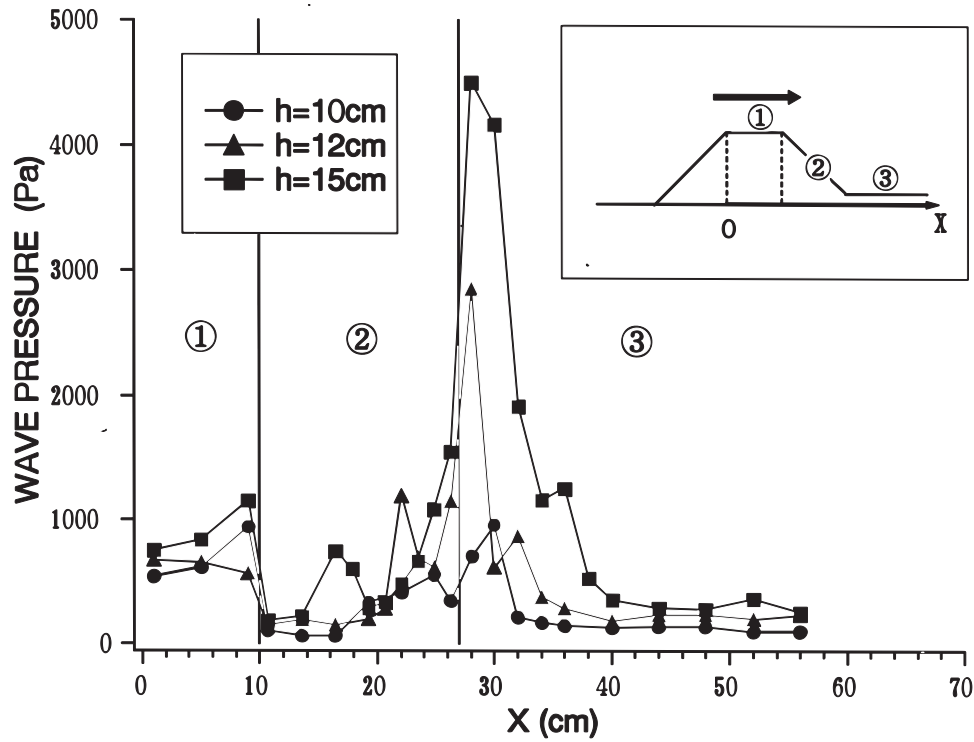
**Figure 3**: Overflowing wave pressure variation with measured point (back slope 1:1).

## 3.   Formulation of Wave Pressures

### 3.1   Dynamic wave pressure

Fukui *et al.* (1962b) empirically propose the following equation to estimate the maximum dynamic wave pressure:

$$\frac{p_{dm}}{\rho_w gh} = K\frac{c^4}{g^2 Hh} \tag{1}$$

where $p_{dm}$ is the maximum kinetic wave pressure, $c$ the wave celerity, $h$ the initial water depth, $H$ the incident wave height, $\rho_w$ the density of seawater, $g$ the acceleration of gravity, and $K$ the kinetic wave coefficient. From the results in this study with Fukui *et al.* (1962b), both values are approximately plotted on the same straight line. Although Fukui *et al.* (1962b) proposed $K = 0.33 \sim 0.51$, it is recommended that $K = 0.12$ be used.

### 3.2   Sustained wave pressure

The maximum sustained wave pressure, $p_{sm}$, can be considered to be related to the maximum dynamic wave pressure, $p_{dm}$, rather than the wave period from the result of this study. The relation between the angle of a sloping
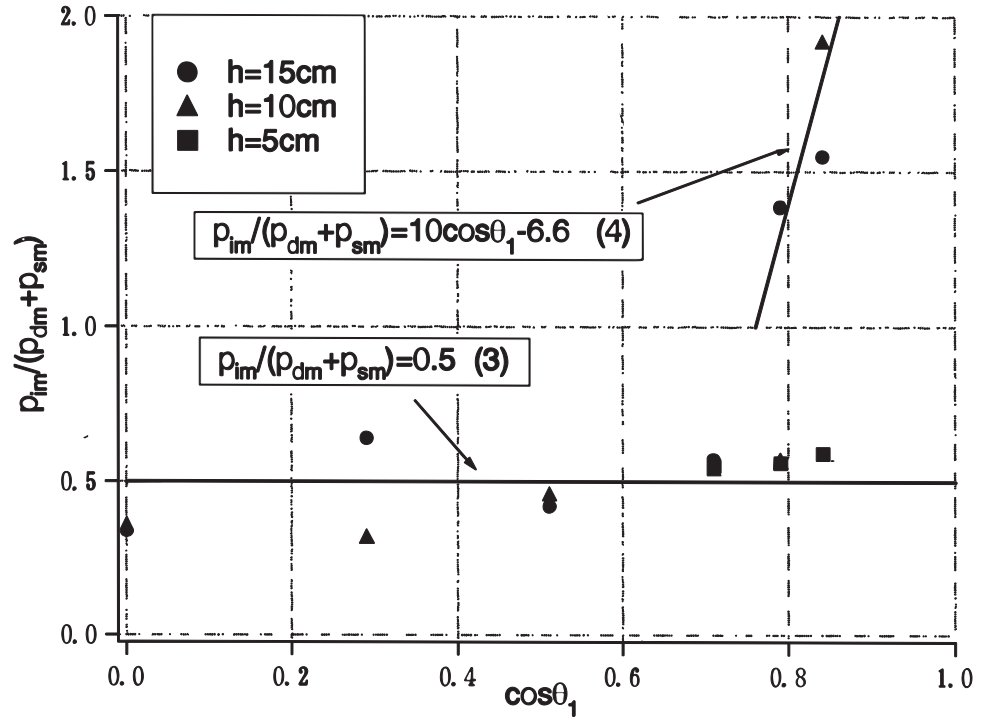
**Figure 4**: Maximum impact standing wave pressure.

board, $\theta_1$, and a non-dimensional value by $p_{dm}$ and $p_{sm}$ suggest the following:

$$\frac{p_{sm}}{p_{dm}} = 0.14(2 + \cos\theta_1)\frac{c^2}{gH} \tag{2}$$

### 3.3  Impact standing wave pressure

Our observations based on using a digital video camera suggest that the impact of standing wave pressure is closely related to the collision of reflected and incident waves, and also that the important parameter for reflected waves is run-up height, and for incident waves is wave celerity. Therefore, a close relationship among run-up height and sustained wave pressure, wave celerity and dynamic wave pressure can be found. The relationship between the sum of maximum dynamic wave pressure, $p_{dm}$, maximum sustained wave pressure, $p_{sm}$, and maximum impact standing wave pressure, $p_{im}$, is shown in Fig. 4. These values have the following relationship:

$$\frac{p_{im}}{p_{dm}+p_{sm}} = 0.5 \quad \left( \frac{g(h+H)\cot\theta_1}{c^2} < 1.1 \right) \tag{3}$$

$$\frac{p_{im}}{p_{dm}+p_{sm}} = 10\cos\theta_1 - 6.6 \quad \left( \begin{array}{c} Fr = \dfrac{c}{\sqrt{g(h+H)}} \leq 1.13 \\[2mm] \dfrac{g(h+H)\cot\theta_1}{c^2} \geq 1.1 \end{array} \right) \tag{4}$$
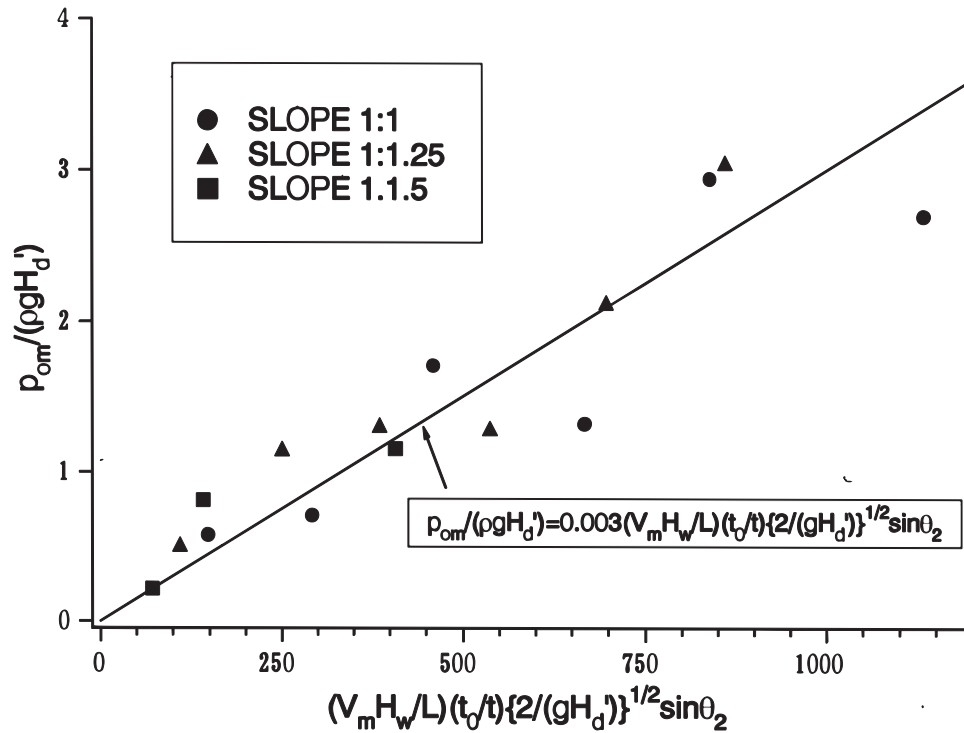
**Figure 5**: Maximum overflowing wave pressure.

Equations (3) and (4) express the result in the case of a larger and smaller slope than 1:1 with different water depth. The difference of (3) and (4) is caused by the collision of reflected and incident waves and its duration. The small collision occurs for a relatively longer time in the case of (3), whereas the large collision takes place for an extremely short time in the case of (4).

## 3.4 Overflowing wave pressure

The maximum overflowing wave pressure occurs when the overflowing wave collides on the back of the structure. It is suggested that the important parameter for the maximum overflowing wave pressure, $p_{om}$, is the maximum velocity, $V_m$, and the water depth on the crest, $H_w$, the angle of the back slope, $\theta_2$, and the height of the models, $H_d'$, and so on. The relationship between the maximum overflowing wave pressure and these parameters is obtained by the law of momentum conservation, and the data are plotted in Fig. 5. These values are satisfied with the following relationship:

$$\frac{p_{om}}{\rho g H_d'} = A \frac{V_m H_w}{L} \frac{t_0}{t} \sqrt{\frac{2}{g H_d'}} \sin \theta_2 \tag{5}$$

where $A$ is the non-dimensional overflowing pressure coefficient, $t_0/t$ the ratio of the time of falling water touched on the bottom after passing the top of the structure, $t_0$, to the duration time acting of the maximum overflowing wave pressure, $t$, $L$ the length acting of the maximum overflowing wave

pressure. The data plotted in Fig. 5 shows that the coefficient $A = 0.003$ is recommended.

## 4.   Conclusions

By using the highly accurate sensor system with the appropriate intervals of time and points, the existence of impact standing and overflowing wave pressure and with a very large value in a short time and at local point can be observed. The value of the impact standing wave pressure due to the collision of the reflected and incident waves is closely related to wave celerity and run-up height. The equations to estimate the maximum values of kinetic, sustained, and impact standing, overflowing wave pressure are proposed in (1)–(5). In the future, we will suppose initial conditions such as the magnitude of the expected tsunami and structures for non-overflowing and overflowing cases, respectively, and will calculate total wave pressures acting on structures using the equations proposed in this study and Goda's formula. Finally, the comparison of two models on their construction will be carried out.

## 5.   References

Fukui, Y., S. Hidehiko, M. Nakamura, and Y. Sasaki (1962a): Study on tsunami. *Coastal Engineering in Japan*, *9*, 44–49 (in Japanese)

Fukui, Y., S. Hidehiko, M. Nakamura, and Y. Sasaki (1962b): Study on tsunami. *Coastal Engineering in Japan*, *9*, 50–54 (in Japanese)

Goda, Y. (1973): A new method of wave pressure calculation for the design of composite breakwaters. *Report of the Port and Harbour Research Institute*, *12*(3).

Matsutomi, H. (1991): The pressure distribution and the total wave force. *Coastal Engineering in Japan*, *38*, 626–630.